CHAPTER 5

METHODOLOGY FOR PREDICTION OF VOLUME CHANGES

5-1. Application of heave predictions

Reasonable estimates of the anticipated vertical and horizontal heave and the differential heave are necessary for the following applications.

a. Determination of adequate designs of structures that will accommodate the differential soil movement without undue distress (chap 6). These predictions are also needed to estimate upward drag from swelling soils on portions of deep foundations such as drilled shafts within the active zone of moisture change and heave. Estimates of upward drag help determine an optimum design of the deep foundation.

b. Determination of techniques to stabilize the foundation and to reduce the anticipated heave (chap 7).

5-2. Factors influencing heave

Table 5-1 describes factors that significantly influence the magnitude and rate of foundation movement. The difficulty of predicting potential heave is complicated beyond these factors by the effect of the type and geometry of foundation, depth of footing, and distribution of load exerted by the footing on the magnitude of the swelling of expansive foundation soil. Additional problems include estimating the exact location that swelling soils will heave or the point source of water seeping into the swelling soil and the final or equilibrium moisture profile in the areas of heaving soil.

Table 5-1. Factors Influencing Magnitude and Rate of Volume Change

Factor	Description
	Soil Properties
Composition of solids	A high percentage of active clay minerals include montmorillonites and mixed layer combinations of montmorillonites and other clay minerals that promote volume change.
Concentration of pore fluid salts	High concentrations of cations in the pore fluid tend to reduce the magnitude of volume change; swell from osmosis can be significant over long periods of time.
Composition of pore fluid	Prevalence of monovalent cations increase shrink-swell; divalent and trivalent cations inhibit shrink-swell.
Dry density	High initial dry densities result in closer particle spacings and larger swells.
Structure	Flocculated particles tend to swell more than dispersed particles; cemented particles tend to reduce swell; fabrics that slake readily may promote swell.
	Environmental Conditions
Climate	Arid climates promote desiccation, while humid climates promote wet soil profiles.
Groundwater	Fluctuating and shallow water tables (less than 20 ft from the ground surface) provide a source of moisture for heave.
Drainage	Poor surface drainage leads to moisture accumulations or ponding.
Vegetative cover	Trees, shrubs, and grasses are conducive to moisture depletion by transpiration; moisture tends to accumulate beneath areas denuded of vegetation.
Confinement	Larger confining pressures reduce swell; cut areas are more likely to swell than filled areas; lateral pressures may not equal vertical overburden pressures.
Field permeability	Fissures can significantly increase permeability and promote faster rates of swell.

5-3. Direction of soil movement

The foundation soil may expand both vertically and laterally. The vertical movement is usually of primary interest, for it is the differential vertical movement that causes most damages to overlying structures.

a. Vertical movement. Methodology for prediction of the potential total vertical heave requires an assumption of the amount of volume change that occurs in the vertical direction. The fraction of volumetric swell N that occurs as heave in the vertical direction depends on the soil fabric and anisotropy. Vertical heave of intact soil with few fissures may account for all of the volumetric swell such that N = 1, while vertical heave of heavily fissured and isotropic soil may be as low as N = 1/3 of the volumetric swell.

b. Lateral movement. Lateral movement is very important in the design of basements and retaining walls. The problem of lateral expansion against basement walls is best managed by minimizing soil volume change using procedures described in chapter 7. Otherwise, the basement wall should be designed to resist lateral earth pressures that approach those given by

$$\sigma_h = K_o\sigma_v \leqslant K_p\sigma_v \qquad (5\text{-}1)$$

where

σ_h = horizontal earth pressure, tons per square root

K_o = lateral coefficient of earth pressure at rest

σ_v = soil vertical or overburden pressure, tons per square foot

K_p = coefficient of passive earth pressure

The K_o that should be used to calculate σ_h is on the order of 1 to 2 in expansive soils and often no greater than 1.3 to 1.6.

5-4. potential total vertical heave

Although considerable effort has been made to develop methodology for reliable predictions within 20 percent of the maximum in situ heave, this degree of accuracy will probably not be consistently demonstrated, particularly in previously undeveloped and untested areas. A desirable reliability is that the predicted potential total vertical heave should not be less than 80 percent of the maximum in situ heave that will eventually occur but should not exceed the maximum in situ heave by more than 20 to 50 percent. Useful predictions of heave of this reliability can often be approached and can bound the in situ maximum levels of heave using the results of both consolidometer swell and soil suction tests described in paragraph 4-2a. The fraction N (para 5-3a) should be 1 for consolidometer swell test results and a minimum of 1/3 for soil suction test results. The soil suction tests tend to provide an upper estimate of the maximum in situ heave (N = 1) in part because the soil suction tests are performed without the horizontal restraint on soil swell that exists in the field and during one-dimensional consolidometer swell tests.

a. Basis of calculation. The potential total vertical heave at the bottom of the foundation, as shown in figure 5-1, is determined by

$$AH = N \cdot DX \sum_{i=NBX}^{i=NEL} DELTA(i)$$

$$= N \cdot DX \sum_{i=NBX}^{i=NEL} \frac{e_f(i) - e_o(i)}{1 + e_o(i)} \qquad (5\text{-}2)$$

where

AH = potential vertical heave at the bottom of the foundation, feet

N = fraction of volumetric swell that occurs as heave in the vertical direction

DX = increment of depth, feet

NEL = total number of elements

NBX = number of nodal point at bottom of the foundation

DELTA(i) = potential volumetric swell of soil element i, fraction

$e_f(i)$ = final void ratio of element i

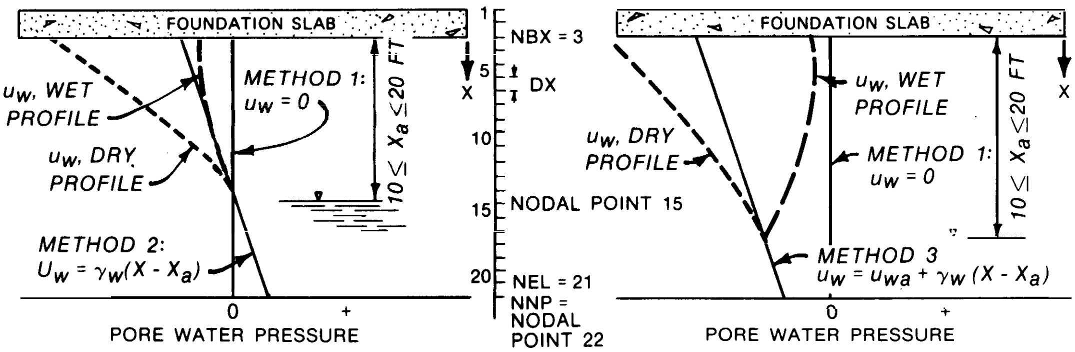
$e_o(i)$ = initial void ratio of element i

The AH is the potential vertical heave beneath a flexible, unrestrained foundation. The bottom nodal point NNP = NEL + 1, and it is often set at the active depth of heave X_a.

(1) The initial void ratio, which depends on geologic and stress history (e.g., maximum past pressure), the soil properties, and environmental conditions shown in table 5-1 may be measured on undisturbed specimens using standard laboratory test procedures. It may also be measured during the laboratory swell tests as described in EM 1110-2-1906. The final void ratio depends on changes in the foundation conditions caused by construction of the structure.

(2) The effects of the field conditions listed in table 5-1 may be roughly simulated by a confinement pressure due to soil and structural loads and an assumption of a particular final or equilibrium pore water pressure profile within an active depth of heave X_a. The effects of confinement and the equilibrium pore water pressure profiles are related to the final void ratio by physical models. Two models based on results of consolidometer swell and soil suction tests are used in this manual (para 4-2a).

b. Pore water pressure profiles. The magnitude of swelling in expansive clay foundation soils depends on the magnitude of change from the initial to the equilibrium or final pore water pressure profile that will be observed to take place in a foundation soil because of



a. Shallow Groundwater Level b. Deep Groundwater Level

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Figure 5-1. Assumed equilibrium pore water pressure profiles beneath foundation slabs.

the construction of the foundation.

(1) *Initial profile.* Figure 5-1 illustrates relative initial dry and wet profiles. The wet initial profile is probably appropriate following the wet season, which tends to occur by spring, while the dry initial profile tends to occur during late summer or early fall. The initial pore water pressure profile does not need to be known if the consolidometer swell model is used because the heave prediction is determined by the difference between the measured initial e_o and final e_1 void ratios (fig. 4-2). The initial void ratio is a function of the initial pore water pressure in the soil. The initial pore water pressure profile, which must be known if the soil suction model is used, may be found by the method described in appendix B.

(2) *Equilibrium profile. The* accuracy of the prediction of the potential total vertical heave in simulating the maximum in situ heave depends heavily on the ability to properly estimate the equilibrium pore water pressure profile. This profile is assumed to ultimately occur beneath the central portion of the foundation. The pore water pressure profile beneath the foundation perimeter will tend to cycle between dry and wet extremes depending on the field environment and availability of water. The three following assumptions are proposed to estimate the equilibrium profile. A fourth possibility, the assumption that the groundwater level rises to the ground surface, is most conservative and not normally recommended as being realistic. The equilibrium profile may also be estimated by a moisture diffusion analysis for steady-state flow, which was used to predict differential heave as part of the procedure developed by the Post-Tensioning Institute (PTI) for design and construction of slabs-on-grade (para 6-3b). The results, which should be roughly compatible with the hydrostatic profiles discussed in *(b)* and (c) below, lead to predictions of heave smaller than the saturated profile.

(a) Saturated. The saturated profile, Method 1 in figure 5-1, assumes that the in situ pore water pressure is zero within the active zone X_a of moisture change and heave

$$u_w = 0 \tag{5-3}$$

where u_w is the pore water pressure in tons per square foot at any depth X in feet within the active zone. Although a pore water pressure profile of zero is not in equilibrium, this profile is considered realistic for most practical cases and includes residences and buildings exposed to watering of perimeter vegetation and possible leaking underground water and sewer lines. Water may also condense in a layer of permeable subgrade soil beneath foundation slabs by transfer of water vapor from air flowing through the cooler subgrade. The accumulated water may penetrate into underlying expansive soil unless drained or protected by a moisture barrier. This profile should be used if' other information on the equilibrium pore water pressure profile is not available.

(b) Hydrostatic I. The hydrostatic I profile, Method 2 in figure 5-la, assumes that the pore water pressure becomes more negative with increasing vertical distance above the groundwater level in proportion to the unit weight of water

$$u_w = \gamma_w(X - X_a) \tag{5-4}$$

where γ_w is the unit weight of water (0.0312 ton per cubic foot).

This profile is believed to be more realistic beneath highways and pavements where drainage is good, pending of surface water is avoided, and leaking underground water lines are not present. This assumption will lead to smaller predictions of heave than the saturated profile of Method 1.

(c) *Hydrostatic II.* This profile, Method 3 in figure 5-lb, is similar to the hydrostatic I profile except that a shallow water table does not exist. The negative pore water pressure of this profile also becomes more negative with increasing vertical distance above the bottom of the active zone X_a in proportion to the unit weight of water

$$u_w = u_{wa} + \gamma_w(X - X_a) \qquad (5\text{-}5)$$

where u_{wa} is the negative pore water pressure in tons per square foot at depth X_a in feet.

(d) Example application. Figure 5-2 illustrates how the saturated (Method 1) and hydrostatic II (Method 3) profiles appear for a suction profile without a shallow water table at a sampling site near Hayes, Kansas. The initial in situ soil suction or negative pore water pressure was calculated from the given natural soil suction without confining pressure τ_o by

$$\tau = \tau^{\circ} - \alpha\sigma_m \qquad (5\text{-}6)$$

where

τ = in situ soil suction, tons per square foot
= compressibility factor (defined in app B)
σ_m = mean normal confining pressure, tons per square foot

The mean normal confining pressure σ_m is given by

$$\sigma_m = \frac{\sigma_v(1 + 2K_T)}{3} \qquad (5\text{-}7)$$

where σ_v is the overburden or vertical confining pressure. The ratio of horizontal to vertical total stress K_T was assumed to be unity. The initial in situ soil suction τ was assumed to be essentially the matrix suction τ_m or negative pore water pressure u_w (i.e., the osmotic component of soil suction τ_s was negligible). The sign convention of the soil suction τ is positive, whereas that of the corresponding negative pore water pressure

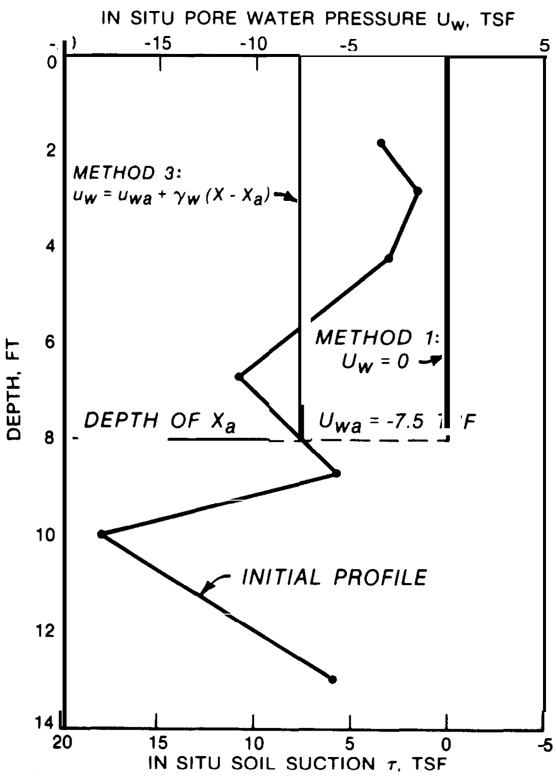

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Figure 5-2. Example application of equilibrium pore water pressure profile for a site near Hayes, Kansas.

u_w is negative (i.e., $\tau_m = -u_w$). Figure 5-2 shows that the hydrostatic equilibrium profile is nearly vertical with respect to the large magnitude of soil suction observed at this site. Heave will be predicted if the saturated profile occurs (Method 1 as in fig. 5-1), while shrinkage will likely be predicted if the hydrostatic II (Method 3) profile occurs. The availability of water to the foundation soil is noted to have an enormous impact on the volume change behavior of the soils. Therefore, the methods of chapter 7 should be used as much as practical to promote and maintain a constant moisture environment in the soil.

c. Depth of the active zone. The active zone depth X_a is defined as the least soil depth above which changes in water content and heave occur because of climate and environmental changes after construction of the foundation.

(1) *Shallow groundwater levels*. The depth X_a may be assumed equal to the depth of the water table for groundwater levels less than 20 feet in clay soil (fig. 5-1a). The u_{wa} term shown in figure 5-1b becomes zero for the hydrostatic I equilibrium profile in the presence of such a shallow water table.

(2) *Deep groundwater levels*. The depth X_a for deep groundwater levels may often be determined by evaluating the initial pore water pressure or suction with depth profile as described in appendix B, The magnitude of u., is then determined after the depth X_a is established.

(a) If depths to groundwater exceed 20 feet beneath the foundation and if no other information is available, the depth X_a can be assumed to be between 10 feet (for moist profiles or soil suctions less than 4 tons per square foot) and 20 feet (for dry profiles or soil suctions greater than 4 tons per square foot) below the base of the foundation (fig. 5-lb). However, the depth X_a should not be estimated less than three times the base diameter of a shaft foundation. Sources of moisture that can cause this active zone include the seepage of surface water down the soil-foundation interface, leaking underground water lines, and seepage from nearby new construction.

(b) The pore water pressure or soil suction is often approximately constant with increasing depth below X_a. Sometimes X_a can be estimated as the depth below which the water content/plastic limit ratio or soil suction is constant.

(c) If the soil suction is not approximately constant with increasing depth below depths of 10 to 20 feet, X_a may be approximated by being set to a depth 1 to 2 feet below the first major change in the magnitude of the soil suction, as shown in figure 5-2.

d. Edge effects. Predictions of seasonal variations in vertical heave from changes in moisture between extreme wet and dry moisture conditions (fig. 5-1) are for perimeter regions of shallow foundations. These calculations require a measure or estimate of both seasonal wet and dry pore water pressure or suction profiles. It should be noted from figure 5-lb that perimeter cyclic movement from extremes in climatic changes can exceed the long-term heave beneath the center of a structure.

(1) *Soil-slab displacements.* A slab constructed on the ground surface of a wet site may in time lead to downwarping at the edges after a long drought or growth of a large tree near the structure (fig. 5-3a). Edge uplift may occur following construction on an initially dry site (fig. 5-3b). The AH in figure 5-3 is representative of the maximum differential vertical heave beneath the slab, excluding effects of restraint from the slab stiffness, but does consider the slab weight.

(2) *Edge distance. The* edge lift-off distance e of lightly loaded thin slabs at the ground surface often varies from 2 to 6 feet but can reach 8 to 10 feet.

(3) *Deflection/length ratio. The* deflection/length ratio of the slab is A/L, where A is the slab deflection and L is the slab length. The angular deflection/span length Δ/ℓ (para 6-1*d*) is twice Δ/L of the slab (fig. 5-3).

e. Methods of predicting vertical heave.

(1) *Hand (manual) applications*. The heave ΔH from a consolidometer test may be found by

$$\frac{\Delta H}{H} = \frac{c_s}{1 + e_o} \log \frac{\sigma_s}{\sigma'_v} \tag{5-8}$$

where

H = thickness of expansive soil layer, feet

c_s = swell index, slope of the curve between points 3 and 4, figure 4-2

σ_s = swell pressure, tons per square foot

σ'_v = final vertical effective pressure, tons per square foot

The final effective pressure is given by

$$\sigma'_v = \sigma_v - u_w \tag{5-9}$$

where σ_v is the total vertical overburden pressure and u_w is the equilibrium pore water pressure in tons per square foot. If u_w is zero for a saturated profile, equation (5-3), then σ'_v is equal to σ_v and heave will be the same as that given by the equation for S_P in figure 4-2. A simple hand method and an example of predicting potential total vertical heave from consolidometer swell tests assuming a saturated equilibrium profile, equation (5-3), are given in TM 5-818-1 and in figure 5-4. However, hand calculations of potential heave can become laborious, particularly in heterogeneous profiles in which a variety of loading conditions need to be evaluated for several different designs,

(2) *Computer applications.* Predictions of potential total heave or settlement can be made quickly with the assistance of the computer program HEAVE available at the U. S. Army Corps of Engineers Waterways Experiment Station. The program HEAVE is applicable to slab, long continuous, and circular shaft foundations. This program considers effects of loading and soil overburden pressures on volume changes, heterogeneous soils, and saturated or hydrostatic equilibrium moisture profiles (equations (5-3) to (5-5)). Results of HEAVE using the saturated profile, equation (5-3), are comparable with results of manual computations described in figure 5-4.

5-5. Potential differential heave

Differential heave results from edge effects beneath a finite covered area, drainage patterns, lateral variations in thickness of the expansive foundation soil, and effects of occupancy. The shape and geometry of the structure also result in differential heave. Examples of effects of occupancy include broken or leaking water and sewer lines, watering of vegetation, and ponding adjacent to the structure. Other causes of differential heave include differences in the distribution of load and the size of footings.

a. *Unpredictability of variables.* Reliable predictions of future potential differential heave are often not possible because of many unpredictable variables that include: future availability of moisture from rainfall and other sources, uncertainty of the exact locations of heaving areas, and effects of human occupancy.

b. Magnitude of differential heave.

(1) Potential differential heave can vary from zero to as much as the total heave. Differential heave is often equal to the estimated total heave for structures supported on isolated spot footings or drilled shafts because some footings or portions of slab foundations often experience no movement. Eventually, differential heave will approach the total heave for most practical cases and should, therefore, be assumed equal to the total potential heave, unless local experience or other information dictates otherwise.

(2) The maximum differential heave beneath a lightly loaded foundation slab may also be estimated by the procedure based on the moisture diffusion theory and soil classification data developed by the PTI. Heave predictions by this method will tend to be less than by assuming that the differential heave is the total potential heave.

5-6. Heave with time

Predictions of heave with time are rarely reliable because the location and time when water is available to the soil cannot be readily foreseen. Local experience has shown that most heave (and the associated structural distress) occurs within 5 to 8 years following construction, but the effects of heave may also not be observed for many years until some change occurs in the

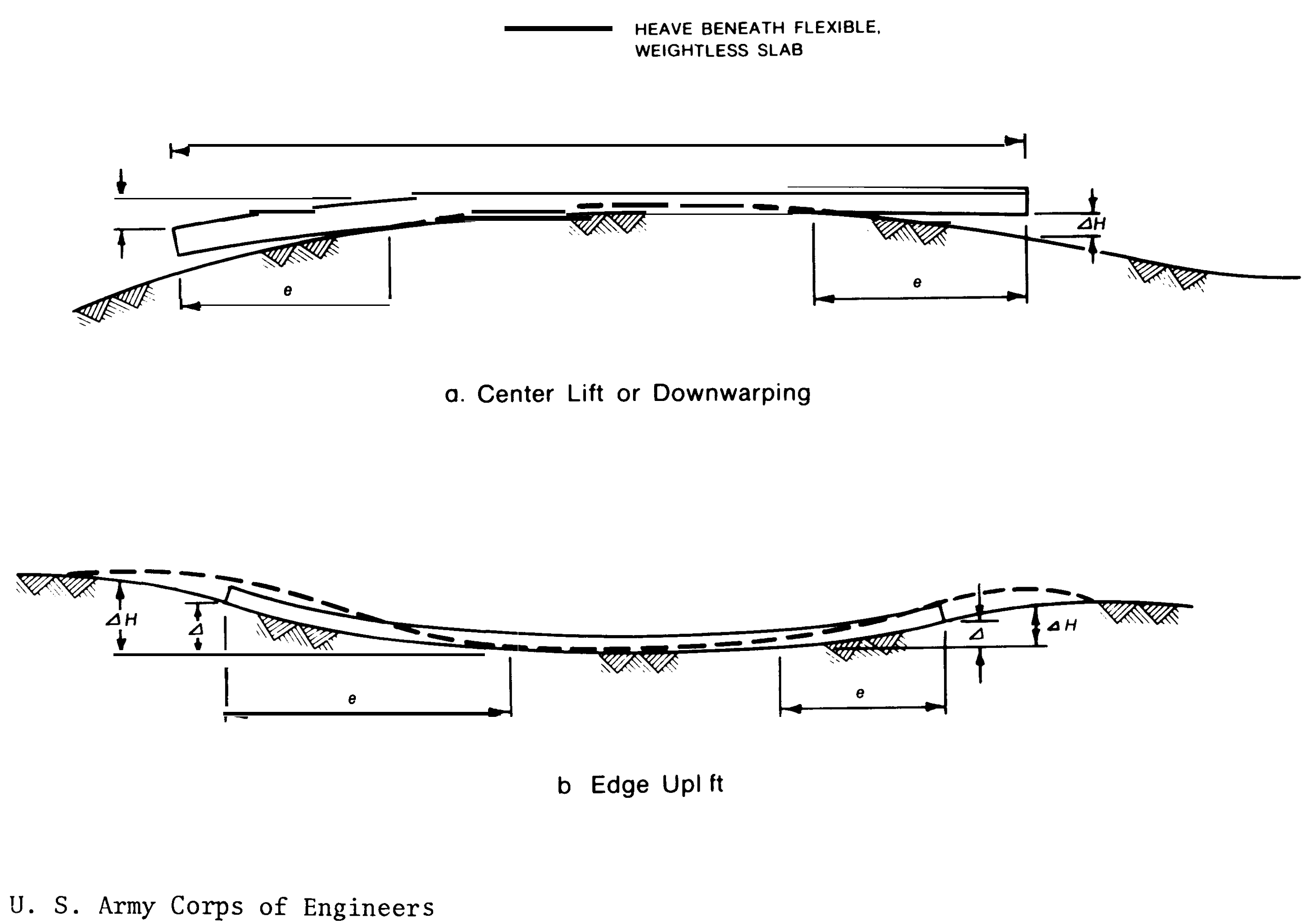
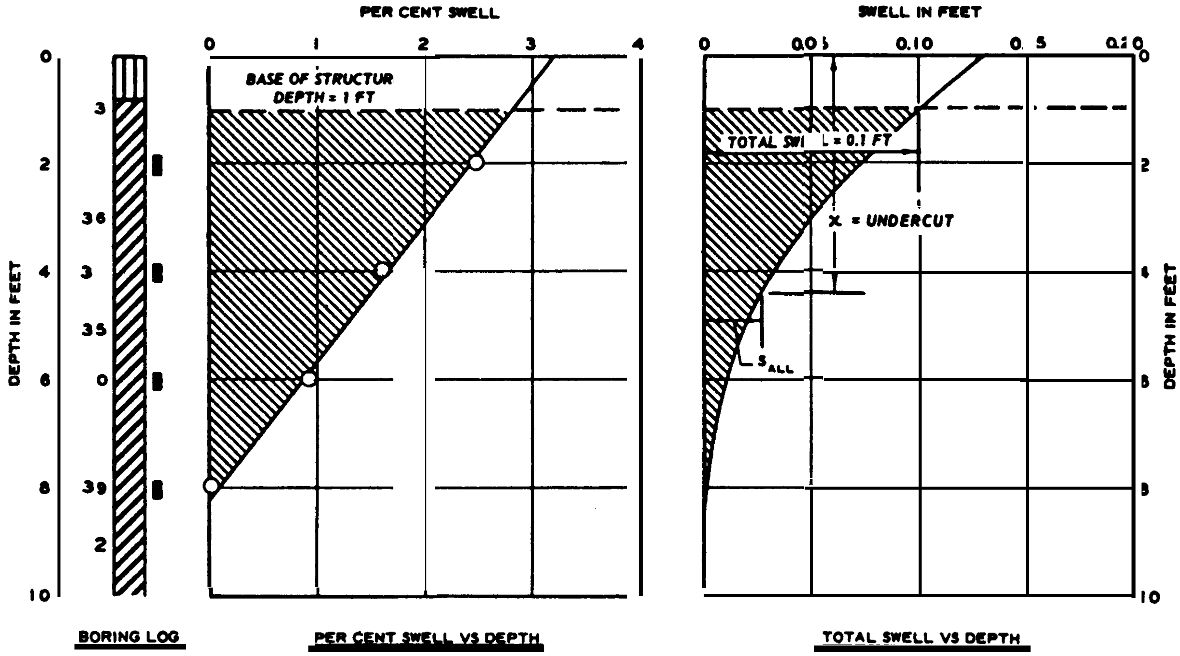


Figure 5-3. Soil-slab displacements on heaving soil.



NOTE: FIGURES TO LEFT OF BORING LOG ARE NATURAL WATER CONTENTS.

▮ DENOTES LOCATION OF SAMPLE FOR SWELL TESTS.

(A) PROCEDURE FOR ESTIMATING TOTAL SWELL

1. ON BASIS OF BORING LOG PROFILE SELECT SAMPLES AT INTERVALS FOR SWELL TESTS.
2. LOAD SPECIMENS IN CONSOLIDOMETER TO OVERBURDEN PRESSURE PLUS WEIGHT OF STRUCTURE; ADD WATER AND OBSERVE SWELL.
3. COMPUTE SWELL IN TERMS OF PER CENT OF ORIGINAL SPECIMEN HEIGHT AND PLOT VS DEPTH.
4. COMPUTE TOTAL SWELL WHICH IS EQUAL TO AREA ENCOMPASSED BY PER CENT SWELL VS DEPTH CURVE. FOR EXAMPLE, USING CURVES SHOWN ABOVE:

TOTAL SWELL = 1/2 × (8.2 − 1.0) × 2.8/100 = 0.10 FT

(B) PROCEDURE FOR ESTIMATING AMOUNT OF UNDERCUT (x) NECESSARY TO REDUCE TOTAL SWELL TO AN ALLOWABLE VALUE (S_{ALL})

1. FROM PER CENT SWELL VS DEPTH RELATIONSHIP, COMPUTE AND PLOT TOTAL SWELL VS DEPTH RELATIONSHIP.
2. FOR A GIVEN VALUE OF S_{ALL}, THE AMOUNT OF UNDERCUT IS READ DIRECTLY OFF THE TOTAL SWELL-DEPTH CURVE.

NOTE: UNDERCUT MATERIAL SHOULD BE REPLACED BY INERT MATERIAL OR ELSE THE BASE OF THE STRUCTURE SHOULD BE LOWERED TO THE DEPTH OF THE REQUIRED UNDERCUT.

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Figure 5-4. Approximate method for computing foundation swell.

foundation conditions to disrupt the moisture regime. Predictions of when heave occurs are of little engineering significance for permanent structures. The important engineering problems are the determination of the magnitude of heave and the development of ways to minimize distress of the structure.